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UPLIFT AND COMPRESSION BEHAVIOUR OF DRILLED PIERS IN LEDA CLAY
PART II - AGED FOOTINGS TESTED

Report No ADDENDUM TO 86-165-K

R.H.K. Tsang
Engineer
Soil Sciences Section
Civil Research Department

ABSTRACT

Full-scale foundation tests were performed as demonstrations for the CIDA-EPCRI Seminar 1219 in 1988. Two "aged" footings were tested in soft, sensitive clay at Gloucester, Ontario, which was the test site of other foundations tested in 1985. A comparative analysis of these "aged" footings with the earlier findings provided insight into the foundation behaviour and responses observed.

job	file	date	report no.
43-301-0812	843.2	October 30, 1991	ADDENDUM TO 86-165-K

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To Mr. D.E. Horrocks
Manager
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ATTN: Mr. H. Dey

UPLIFT AND COMPRESSION BEHAVIOUR OF DRILLED PIERS IN LEDA CLAY
PART II - AGED FOOTINGS TESTED

1.0 INTRODUCTION

The Soil Sciences Section, Civil Research Department, hosted the CIDA-EPCRI Mission 1219, entitled "Transmission Tower Foundations Studies", August 29 to September 9, 1988 Appendix A)/1/. As part of the seminar, full-scale field load testing and in situ soil testing demonstrations were performed by Ontario Hydro and the National Research Council staff. These demonstrations were performed at the Gloucester test site near Ottawa. This demonstration site was selected because it reflected the problems encountered in constructing a 500 kV transmission line in soft, sensitive clay in 1985.

2.0 PREVIOUS FULL-SCALE FOUNDATION TESTING PROGRAM

In 1985, proposed 500 kV transmission lines, namely, Lennox GS to Hawthorne TS, Hawthorne TS to Merivale TS, and Merivale TS to Cornwall TS, were to traverse through areas covered with deep deposits of soft, sensitive clay. This clay deposit, commonly known as Leda clay, was formed in the Champlain Sea during a single marine episode which ended as a result of isostatic rebound. These sediments were deposited initially in estuarine conditions, and then, uplifted into a fresh water, lacustrine environment.

These clays exhibit a high "sensitivity" resulting in a significant loss of shear strength due to soil disturbance and/or remoulding. Any physical disturbance of the clay structure can turn the soil mass into a viscous fluid.

Three features that characterize the unusual behaviour of Leda clay are; (1) the possibility of excessive settlements due to the open nature of the soil fabric, (2) the possibility of substantial loss of strength resulting from the breakdown of the soil structure, and (3) the possibility that the lost strength cannot be recovered through thixotropic processes due to the breakdown of the structure/2/.

Due to the unique behaviour of this soft, sensitive clay (Leda clay), an extensive full-scale testing and construction program for drilled piers was performed at Gloucester, Ontario in 1985. The details of the construction and excavation methods are documented elsewhere/2/. Test Footing No 5 was not tested in 1985 but allowed to "age" to investigate time-dependent effects and behaviour. In 1988 with the CIDA-EPCRI Mission 1219, an opportunity arose to perform full-scale load tests on "aged" footings. The many departments and organizations participating in this seminar are acknowledged for their valuable contributions.

The original test program (1985) addresses the following questions/2,4/:

1. Constructability of large diameter (up to 2.7 m) and deep (up to 13 m) augered pier foundations in a soft Leda clay deposit.

2. Ultimate load and load-deformation behaviour of augered pier foundations in Leda clay under the type of loads induced by 500 kV lattice type transmission towers.

A total of 8 test footings, having a nominal size of 1.53 m in diameter and 6.4 m in depth, were installed. The following aspects related to the foundation performance were investigated:

1. Ultimate load and load-deformation behaviour under short-term, uplift loads.

2. Effect of cyclic loading on uplift capacity.

3. Effect of construction methods on uplift capacity.

4. Thixotropic gain or development of the bond between the concrete footing and the clay with time.

5. Effect due to rate of loading on the uplift capacity.

6. Ultimate load and load-deformation behaviour under short-term compression loads.

7. Load-deformation behaviour under a ground line shear force.

Table I summarizes details of the footing dimensions for the various test footings. Table II shows the foundation testing sequence in 1985, and in 1988 respectively.

3.0 FULL-SCALE FOUNDATION TESTING OF "AGED" FOOTINGS

The results of the full-scale testing in 1988 on Footings 4 and 5 (Tests 10 and 11) are reported. An analysis of these two "aged" Footings 4 and 5 are compared to the 1985

foundation full-scale tests is presented. Further insight to the foundation behaviour of these "aged" footings (approximately 38 weeks) is examined and documented.

3.1 Observations from the 1985 Test Program

The site layout at Gloucester shows the locations of Footings 4 and 5 respectively (Figure 1). Figure 2 shows the detailed soil profile for the test site/4,5/.

Figure 3 depicts the typical load-deflection curve for Footing 9, including pore water pressure measurements, tested after 2 weeks since construction in 1985. Note the large suction pressures (-80 kPa) generated at the base of the footing due to the uplift loading at large displacements. However, relatively small negative pore water pressures (-5 kPa) were generated in the working load range with small displacements (2 mm).

Figures 4 and 5 depict the load-deflection and pore water pressure curves, respectively, for Footing 4 under sustained or long-term uplift loading. Note the large negative pore water pressures (-40 kPa) generated at large footing displacements. Again, small footing displacements (2 mm) correspond to small suction pressures of -10 kPa being observed.

The full-scale tests in 1985 for Footings 9 and 4 (long-term) are reported, herein, for comparison purposes with the full-scale tests performed in 1988. Both Footings 4 and 5 were tested in short-term or rapid uplift. Footing 4, though, was retested in 1988 to identify any changes in capacity and performance from 1985.

3.2 Observations from the 1988 Test Program

Details of the instrumentation layout for Footings 4 and 5 are contained in Appendix B. The details of the full-scale testing methods for these tests are reported elsewhere/2,3/. The full-scale load testing data, load, displacement, and pore water pressure, are reported in Appendix B. In addition, the National Research Council performed piezocone testing at the Gloucester site for seminar demonstration purposes in 1988. The undrained shear strength profile is presented in Appendix C.

The piezocone estimated the undrained shear strength, Su, to range from 20 to 25 kPa for a depth of 2 m to 10 m in the range of footing embedment lengths. Overall, the piezocone undrained shear strength profile compares well to the strength profile in Figure 2, for a depth of 0 to 20 m using the dilatometer, the NGI vane, and CIU laboratory testing. The dilatometer appears consistently to underestimate Su by the vane and CIU. The piezocone Su profile appears consistent and within the range of these tests over the same depth interval.

Figure 6 shows the 1988 load-displacement curve for Footing No 5 aged after 38 weeks. The behaviour of this footing is similar to Footing No 9 (tested 2 weeks after construction in 1985).

Figure 7 shows the 1988 load-displacement curve for Footing No 4 aged after 38 weeks. The behaviour of this retested footing differs from the initial sustained or long-term uplift test in 1985. Note the large negative pore water pressures (-90 kPa) generated at the base of Footing No 4 tested in uplift (Figure 7). This pore water response for Footing No 4 retested in uplift (aged 38 weeks) is similar to Footing No 9 (aged 2 weeks) and differs from the long-term sustained uplift load test (aged 13 weeks) (Figures 3 to 5).

Table III summarizes the uplift test results for 1985, and 1988 for comparison purposes. The uplift tests performed in 1988 did not appear to increase in average adhesion between the Leda clay and concrete after being "aged" 38 weeks. The range in adhesion varied from 10 kPa to 19 kPa for the different footings tested in 1985, with exception of the long-term or sustained loading on Footing No 4.

4.0 DISCUSSION OF OBSERVATIONS

The following discussion of the 1988 results supplements the findings of the previous foundation testing program in 1985:

1) Similar load-displacement curves are observed for Footings 5, 9, 10, and 11. The loads increased rapidly to near maximum values with footing uplift displacements less than 5 mm. All the observed curves had a distinct "knee" in the vicinity of 5 mm displacement, followed by subsequent pullout dominated by creep effects.

2) The exception to the "brittle or sudden" manner of failure was retesting of Footing 4 (1988) in uplift. The observed load-displacement curve is more non-linear in behaviour. This might be expected since the original adhesion of clay to concrete was broken from the long-term or sustained loading in 1985 on Footing 4 (Figures 4, 5 and 7).

3) The 1988 uplift load tests for Footing 4 (retest) and Footing 5 was analyzed in a similar manner to 1985. In each load test, the contributions of the footing weight and the frictional resistance from the upper granular layers were subtracted from the measured ultimate load to obtain the adhesion resistance contributed by the clay layer (Table III). The frictional resistance was calculated using the Ku theory for the gravel and sandy silt layer/2/. The average adhesion over the embedded footing length in clay was calculated by dividing the clay adhesion resistance by the surface area of the footing in contact with the clay layer.

4) This average adhesion value uses the measured ultimate loading which corresponds to large uplift displacements (> >25 mm) and at high load levels with significant foundation creep.

5) The average adhesion values of the footing embedded in clay observed from Table III ranges from 8.9 kPa to 18.9 kPa/2/. The two tests performed in 1988 for Footing 5 and retesting of Footing 4 resulted in average adhesion values of 17.4 kPa and 14.2 kPa respectively. Both these adhesion values are within the range previously observed in 1985, even after aging 38 weeks.

6) It should be noted that Footing 4 in 1985 was tested in long-term sustained loading, and in 1988 was retested in short-term or rapid uplift. Two different average adhesion values of the footing embedded in clay were obtained, 8.9 kPa (under sustained uplift loading in 1985) and 14.2 kPa (under uplift loading in 1988). It would appear that the footing resistance had gained or regained strength after 38 weeks. Two different uplift tests were performed, namely, a long-term sustained uplift test and a short-term uplift test. The apparent increase in footing resistance may not be a regain in strength of the failed clay but rather differences in the type of uplift test performed. This is reflected in the two distinctly different load-displacement curves observed (Figures 4 and 7). These two tests for Footing 4 differ from the other footings tested as discussed in #1 and #2 above.

7) Comparison of Footing 9 (youngest footing tested after 2 weeks in 1985) and Footing 5 (aged 38 weeks to 1988) did not appear significantly different. The average adhesion values over the embedded length in clay were 18.9 kPa (Footing 9) and 17.4 kPa (Footing 5) respectively. No significant effect due to aging was observed. Construction of Footings 4 and 5 resulted in minor soil disturbance at the base of the footing, and the manner of construction varied from the other footings (Table I)/2/. The minor soil disturbance or manner of construction did not alter significantly the results for Footing 5 in 1988.

8) The pore water pressures were monitored in Footing 4 to observe the response due to uplift load testing. The maximum decrease in pore water pressure observed during the long-term sustained uplift load test (1985) was -40 kPa (Figures 4 and 5). The maximum decrease in pore water pressure observed during the short-term uplift test (1988) was -90 kPa (Figure 7). These pore water pressure measurements were recorded at the base of the footing at depth 7.32 m. Another IRAD vibrating wire piezometer, that was installed at a depth of 3.70 m on the outside edge (15 cm) of the footing, observed no changes in pore water pressures from ambient. The large negative pore water pressure (-90 kPa) indicated a large temporary suctional force (636 kN) developed during the short-term uplift loading. This large suctional force appears greater than the measured ultimate load less the footing weight (ie the soil friction and adhesion resistance). Note the negative pore water pressure observed in the short-term uplift test (1988) dissipated after 19 hours, returning to initial ambient pressure (Figure 7). Correspondingly, the large temporary suctional force developed during maximum uplift loading and displacements, also, dissipated after unloading. In the case of the long-term sustained uplift test (1985), both the negative pore water pressure and temporary suction developed during the 1985 test was less

in magnitude than the short-term uplift test (1988). The differences in the pore water pressure response observed for Footing 4 (1985 and 1988) were due to the different uplift tests performed, namely, long-term sustained and short-term uplift. Under long-term sustained loadings, the induced pore water pressure had sufficient time to dissipate during this "slow" test in clay.

The EPRI TL Work Station, CUFAD version 1.0, compression/uplift foundation analysis and design program, can include the "undrained tip/base suction" as a component to estimate the footing capacity/6/. However, based on the observed load tests, this large suctional force is only temporary which dissipates with time and occurs at correspondingly large uplift displacements (>>25 mm). At working load levels, the small suctional forces developed at small uplift displacements (<5 mm); negative pore water pressures observed were -5 kPa to -10 kPa. Therefore, the basal suction does not increase significantly the foundation capacity under rapid short-term loading, and even less under long-term sustained loading especially in the working load range. For these reasons, the suction force at the footing base should not be included in design estimates of foundation capacity.

9) Excluding the long-term sustained uplift test in 1985 (Footing 4), the average adhesion value for all the uplift tests performed was 15.0 kPa ranging from 10 kPa to 19 kPa (Table III). The undrained shear strength, obtained from the dilatometer and piezocone, ranged from 20 kPa to 25 kPa over a depth of 2 m to 10 m (Figure 2 and Appendix C). The average undrained shear strength of 22.5 kPa estimated by the dilatometer and piezocone is representative for this site. The adhesion factor, α, is 0.67 determined by dividing the average adhesion value of the footing embedded in clay with the average undrained shear strength. The adhesion factor α, is used in design calculations to estimate the footing uplift capacity in clay. This is consistent with the 1986 recommended α factor of 0.65 applied to the average in situ undrained shear strength to determine shaft resistance/2/.

5.0 CONCLUSION

The opportunity to perform full-scale foundation tests, as demonstrations for CIDA-EPCRI Seminar 1219, enhanced Ontario Hydro's experience and database of drilled piers constructed in soft, sensitive clay. The results of the full-scale load tests performed in 1988 are documented. A comparative analysis of two recent tests on "aged" footings with the original foundation test program was performed. Further insight of the foundation behaviour and response for these "aged" footings is reported.

6.0 RECOMMENDATIONS

The following summary reflects the major findings of this full-scale foundation test program for drilled piers in soft, sensitive clay:

(a) Similar foundation responses for "aged" footings (38 weeks) and "young" footings (2 weeks) were observed, as indicated by the load-displacement curves.

(b) In the soft, sensitive clays, the drilled piers exhibited considerable creep effects and a "brittle" manner of foundation failure, typically less than 5 mm, during the different pullout tests performed.

(c) No apparent increase in footing resistance or capacity resulted due to aging even testing after 38 weeks since construction.

(d) The type of full-scale uplift test performed affects the foundation response, capacity, and displacements. Several different uplift tests were performed in this program such as short-term uplift test, long-term sustained uplift test, cyclic uplift test, and combined uplift and lateral tests.

(e) Large negative pore water pressures were observed during the full-scale uplift tests only at large footing displacements ($>>25$ mm). A large suctional force at the base of the footing developed. However, this temporary suctional force reduces as the negative pore water pressure dissipates in the clay. The suctional force should not be included in the design estimates of foundation capacity as used in CUFAD by EPRI TL Work Stations. At the working load level, small footing displacements (<2 mm) resulted only in negative pore water pressures of -5 kPa to -10 kPa.

(f) The 1986 recommended adhesion factor, α, of 0.65 for soft, sensitive Leda clay was based on the full-scale tests performed. This adhesion factor is confirmed by the recent tests performed in 1988.

(g) In situ penetration testing in soft sensitive clay using the piezocone and dilatometer provided reasonable estimates of undrained shear strength.

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2) The seminar was hosted by the Soil Sciences Section, Civil Research Department. The following participated in presentations and discussions chaired by Dr. T.W. Klym, Manager, Civil Research Department; Mr. T.J. Carmichael, Dr. H.S. Radhakrishna, Mr. C.B. Cragg, Dr. H.T. Chan, Mr. A. Vorauer, and Dr. K.C. Lau assisted in Chinese translations. The Mechanical Research Department participated in presentations and discussions by Mr. C. Pon and Mr. G. Senkiw. Civil Engineering and Architecture, Transmission Lines Programs, with assistance from Mr. H. Dey and Mr. T. Tong discussed design practices within Ontario Hydro. Mr. G. Aitchison, Lines Superintendent, Lines and Stations Construction, assisted in field tours of transmission tower line construction and stringing operations. Mr. K.K. Subbayya, New Business Ventures Division, assisted in coordinating this seminar.

3) The National Capital Commission and the Department of National Defence for permitting access to the test site at the Gloucester Naval Base.

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5) Messrs D. Fraser, Soil Sciences Section, D. Auston and D. King, Mechanical Research Department, assisted during the full-scale load testing.

6) The Lines and Stations Construction staff assisted during load testing.

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4. Radhakrishna, H.S., Cragg, C.B.H., R. Tsang and M. Bozozuk. "Uplift and Compression Behaviour of Drilled Piers in Leda Clay". Proceedings from the 39th Canadian Geotechnical Conference In Situ Testing and Field Behaviour, Ottawa, Ontario, August 1986.

5. Bechai, M., Law, K.T., C.B.H. Cragg and Konrad, J.M. In Situ Testing of Marine Clay for Tower Line Foundations. Proceedings from the 39th Canadian Geotechnical Conference, In Situ Testing and Field Behaviour, Ottawa, Ontario, August 1986.

6. Seminar entitled "Foundation Design for Transmission Line Structures". Attended by R.H.K. Tsang, presented by F.H. Kulhawy, C.H. Trautmann of Cornell University and A.M. DiGioia, Jr., and R.A. Braggs of GAI Consultants Inc. and sponsored by Electrical Power Research Institute, January 1986.

TABLE I

Summary of Footing Dimensions as Constructed

Footing Number	Steel Liner: Embedded Length (m)	Steel Liner: Projection Above Gravel Fill (m)	Footing Diameter: Cased Portion (m)	Footing Diameter: Uncased (i) Augered Portion (m)	Measured Augered Length (m)	Projection of Footing Above Gravel Fill (m)	Type of Load Tests
1+	2.59	0.76	0.94	0.91	9.65	0.30	uplift
2	1.83	0.30	1.55	1.45	6.40	0.46	compression
3	1.98	0.15	1.55	1.45	6.40	0.30	uplift
4+,*	1.45	0.69	1.55	1.45	6.81	0.30	uplift
5+,*	1.52	0.61	1.55	1.45	6.32	0.30	uplift
6	2.92	1.35	2.53	2.44	6.63	3.35	moment
7	3.20	1.37	1.70	1.50	6.93	3.33	moment
8!	2.79	1.47	2.79	2.72	12.73	3.35	moment
9	4.04	0.53	1.70	1.50	6.88	0.61	uplift
10	3.96	0.61	1.70	1.50	6.71	0.61	uplift
11	3.71	0.86	1.70	1.50	6.65	0.61	uplift

Additional Notes: All of the footings constructed in the dry with exceptions as noted:

(+) noticed soil disturbance at bottom of augered hole
(*) water added to support hole (4.27 m deep)
(!) minor caving at side of hole below liner after 5 hours being left open and reinforcing cage sank to depth of 12.95 m
(i) auger diameter plus estimated "auger whip" of 2.5 to 5 cm

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TABLE II

Part (A)
Foundation Testing Sequence
in 1985 (Oct. 3 - Nov. 1)

Test No	Footing No	Test Type	Test Details	Footing Age (weeks)
1	9	uplift	effect of age	2
2	11	uplift	cycled loads	5
3	2	compression		12
4	1	uplift	effect of scale	12
5	3	combined	lateral & uplift loads	12
6	2 & 3	lateral		12
7	10	uplift	sand & gravel layers removed	5
8	1 & 2	lateral	scale effect	13
9	4	uplift	long term loads	13
not tested	5	uplift		

Part (B)
Foundation Testing Sequence
in 1988 (Sept. 8 - 9)

Test No	Footing No	Test Type	Test Details	Footing Age (weeks)
10	4	retest uplift (Test No 9)	effect of age and retest	~38
11	5	uplift	effect of age	~38

TABLE III

Summary of Uplift Test Results

Footing Number	Depth (m)	Diameter (m)	Total Length (m)	Weight Ftg (kN)	Contact Area (m^2)		Ultimate Load (kN)	Ultimate Capacity Contributed From Gravel Pad and Silty Sand (kN)	Ultimate Capacity Contributed From Leda Clay (kN)	Average Adhesion of Leda Clay to Concrete (kPa)
					Gravel and Silty Sand	Clay				
uplift 1	0-2.59 2.59-9.73	0.9398 0.9144	10.03	150.50	4.95	23.20	444.9 (100 kips)	57.4 (13 kips)	237.0 (53 kips)	10.2 (213 psf)
lateral & uplift 3	0-1.98 1.98-6.48	1.5494 1.4478	6.78	263.32	8.16	21.93	694.2 (156 kips)	93.7 (21 kips)	337.1 (76 kips)	15.4 (321 psf)
long term 4	0-1.45 1.45-6.88	1.5494 1.4478	7.19	275.52	8.16	25.47	533.9 (130 kips)	93.7 (21 kips)	164.7 (47 kips)	8.9 (185 psf)
uplift 9	0-4.04 4.04-6.96	1.7018 1.4986	7.57	347.90	8.96	26.38	947.8 (213 kips)	101.9 (23 kips)	497.9 (112 kips)	18.9 (394 psf)
excavated sand 10	0-3.96 3.96-6.78	1.7018 1.4986	7.39	340.00	0.00	23.05	654.2 (147 kips)	0.00	314.2 (70 kips)	13.6 (285 psf)
cyclic 11	0-3.71 3.71-6.73	1.7018 1.4986	7.34	335.07	8.96	25.09	827.7 (185 kips)	101.9 (23 kips)	390.6 (88 kips)	15.6 (325 psf)
*uplift 5	0-1.45 1.45-6.32	1.5494 1.4478	6.62	253.68	8.16	23.45	756.5 (170 kips)	93.7 (21 kips)	409.1 (92 kips)	17.4 (363 psf)
*retest uplift 4	0-1.45 1.45-6.32	1.5494 1.4478	7.19	275.52	8.16	25.47	729.8 (164 kips)	93.7 (21 kips)	360.58 (81 kips)	14.2 (296 psf)

Note: (*) Tested in 1988, all others tested in 1985

ADDENDUM TO 86-165 Rev 3-K20

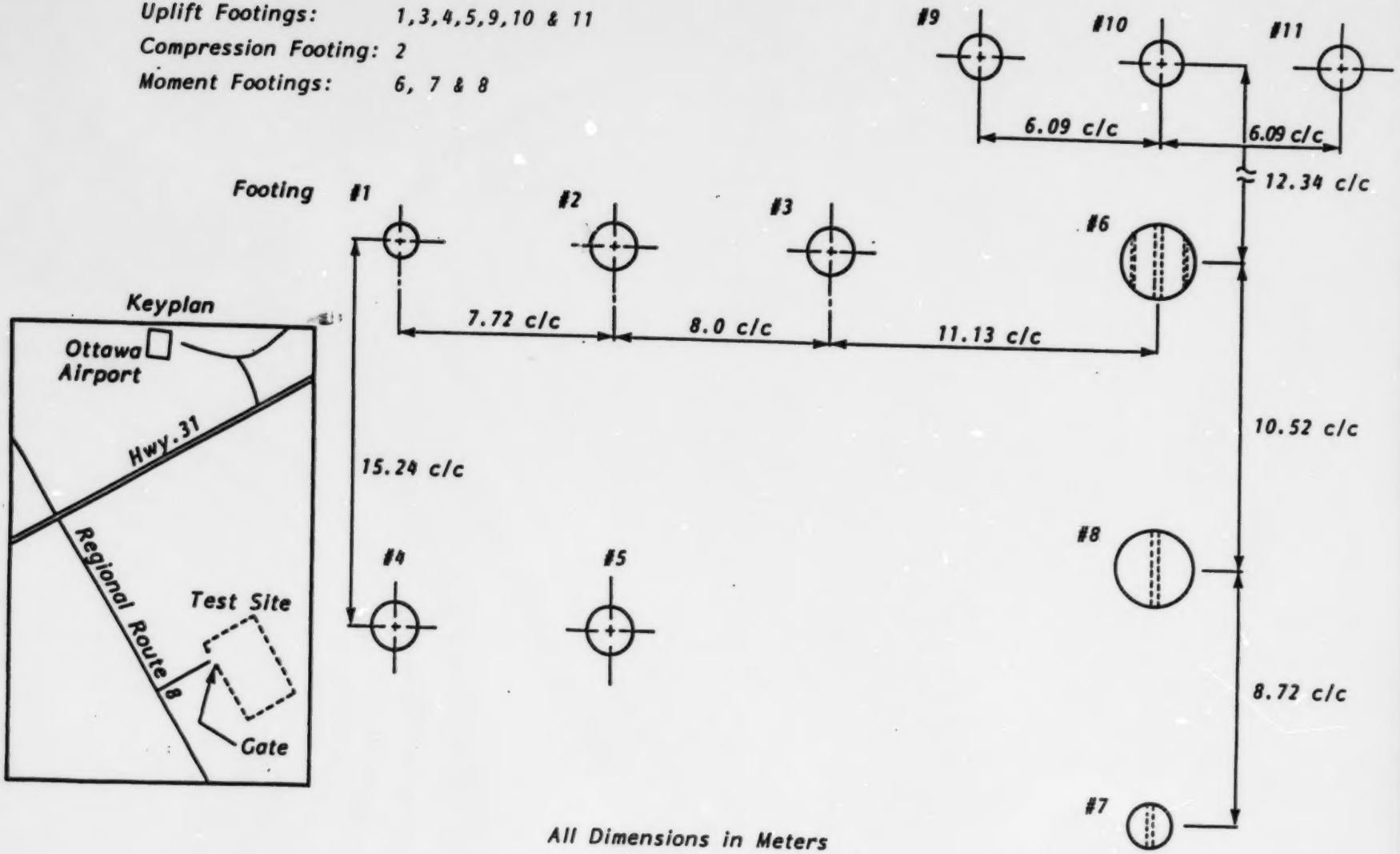


FIGURE 1
LAYOUT OF FOUNDATION FOOTINGS
AT GLOUCESTER TEST SITE

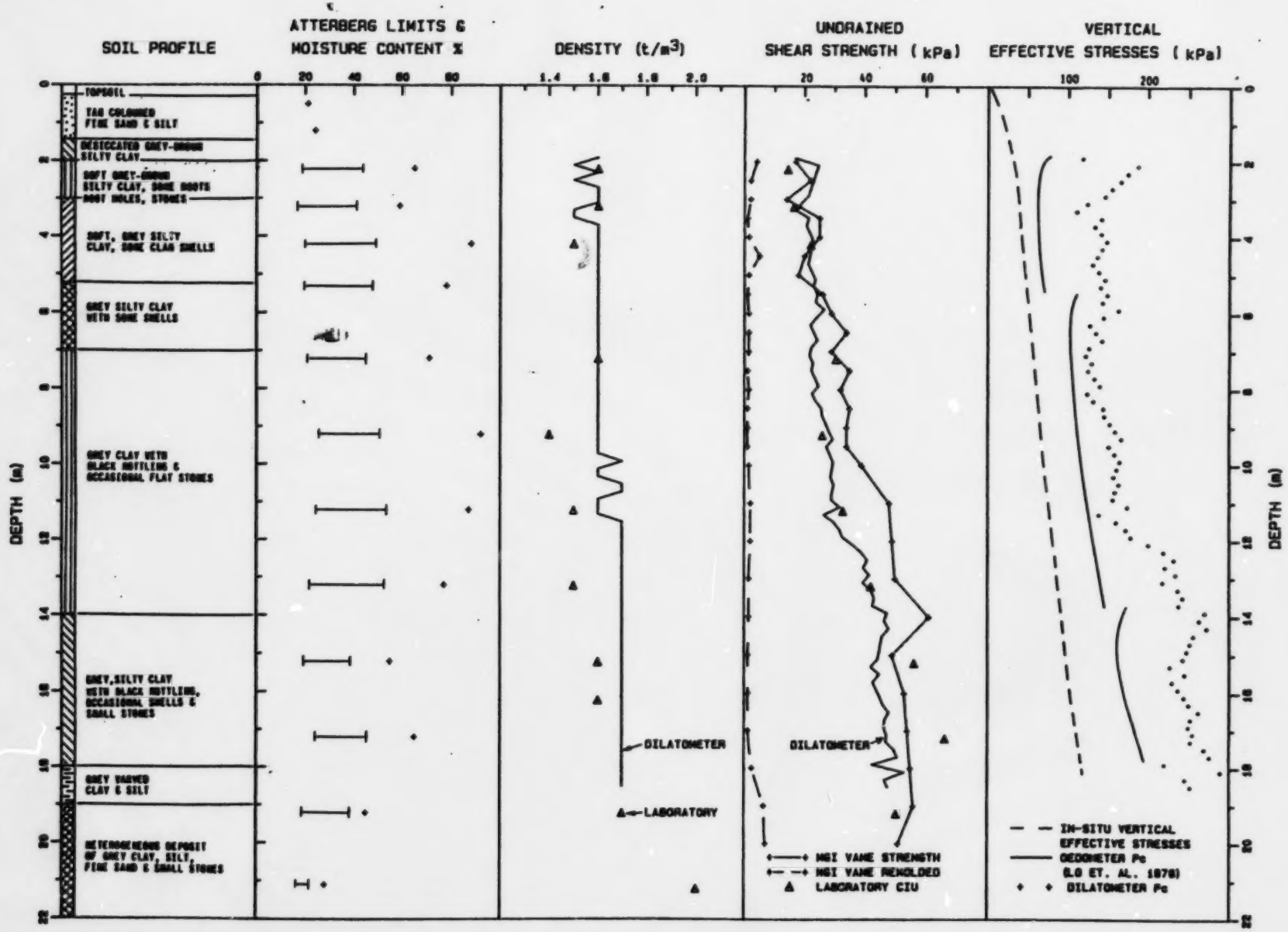

SOIL PROFILE
ATTERBERG LIMITS & MOISTURE CONTENT %
DENSITY (t/m3)
UNDRAINED SHEAR STRENGTH (kPa)
VERTICAL EFFECTIVE STRESSES (kPa)
TOPSOIL
TAN COLOURED FINE SAND & SILT
DESICCATED GREY-BROWN SILTY CLAY
SOFT GREY-BROWN SILTY CLAY, SOME ROOTS
ROOT HOLES, STONES
SOFT, GREY SILTY CLAY, SOME CLAM SHELLS
GREY SILTY CLAY WITH SOME SHELLS
GREY CLAY WITH BLACK MOTTLING & OCCASIONAL FLAT STONES
GREY, SILTY CLAY WITH BLACK MOTTLING, OCCASIONAL SHELLS & SMALL STONES
GREY VARVED CLAY & SILT
HETEROGENEOUS DEPOSIT OF GREY CLAY, SILT, FINE SAND & SMALL STONES
DEPTH (m)
DILATOMETER
LABORATORY
NGI VANE STRENGTH
NGI VANE REMOLDED
LABORATORY CIU
IN-SITU VERTICAL EFFECTIVE STRESSES
OEDOMETER Pc
(LO ET. AL. 1978)
DILATOMETER Pc

FIGURE 2

SOIL PROFILE FOR TEST SITE AT GLOUCESTER

ADDENDUM TO 71477 RD
86-165

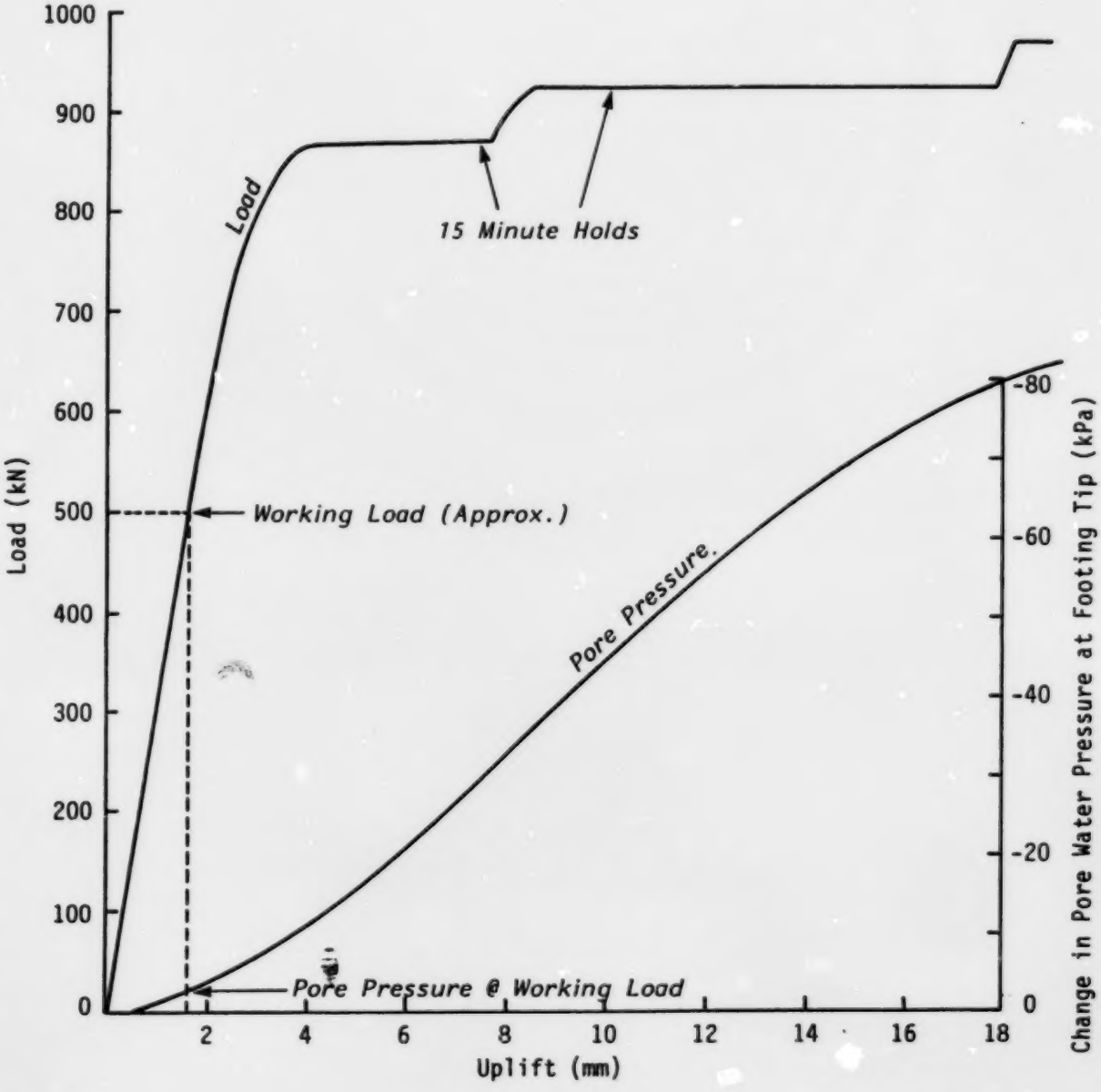


FIGURE 3

UPLIFT TEST ON FOOTING #9 IN 1985
(Youngest Footing 2 Weeks)

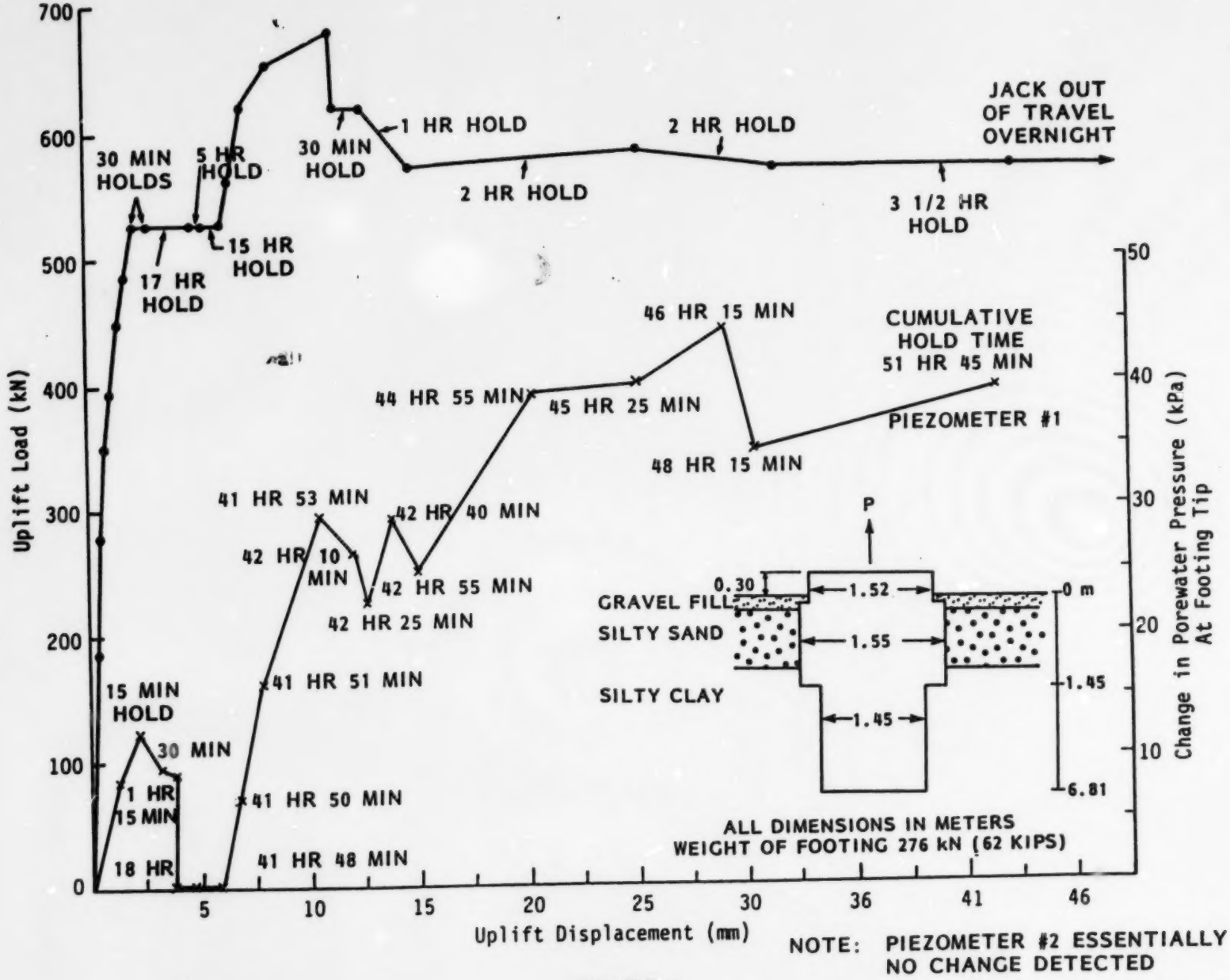


FIGURE 4

LONG TERM TEST ON FOOTING #4 IN 1985

ADDENDUM TO
86-165
177298 RD

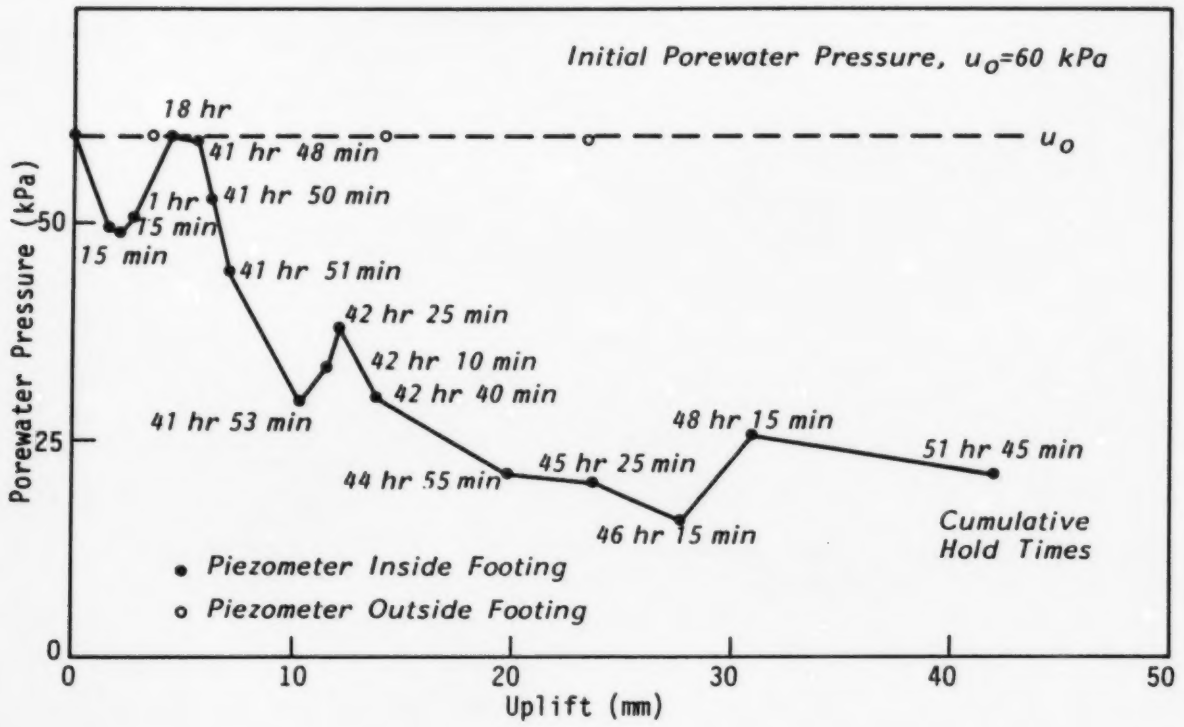


FIGURE 5

POREWATER PRESSURE FOR LONG TERM UPLIFT ON FOOTING #4 IN 1985

Test No. 9
Ground Elevation 29.50m
Elevation of Piezometer Inside Footing 22.49m
Elevation of Piezometer Outside Footing 26.10m
(at 15cm from edge of footing)
Elevation of Footing Base 22.69m

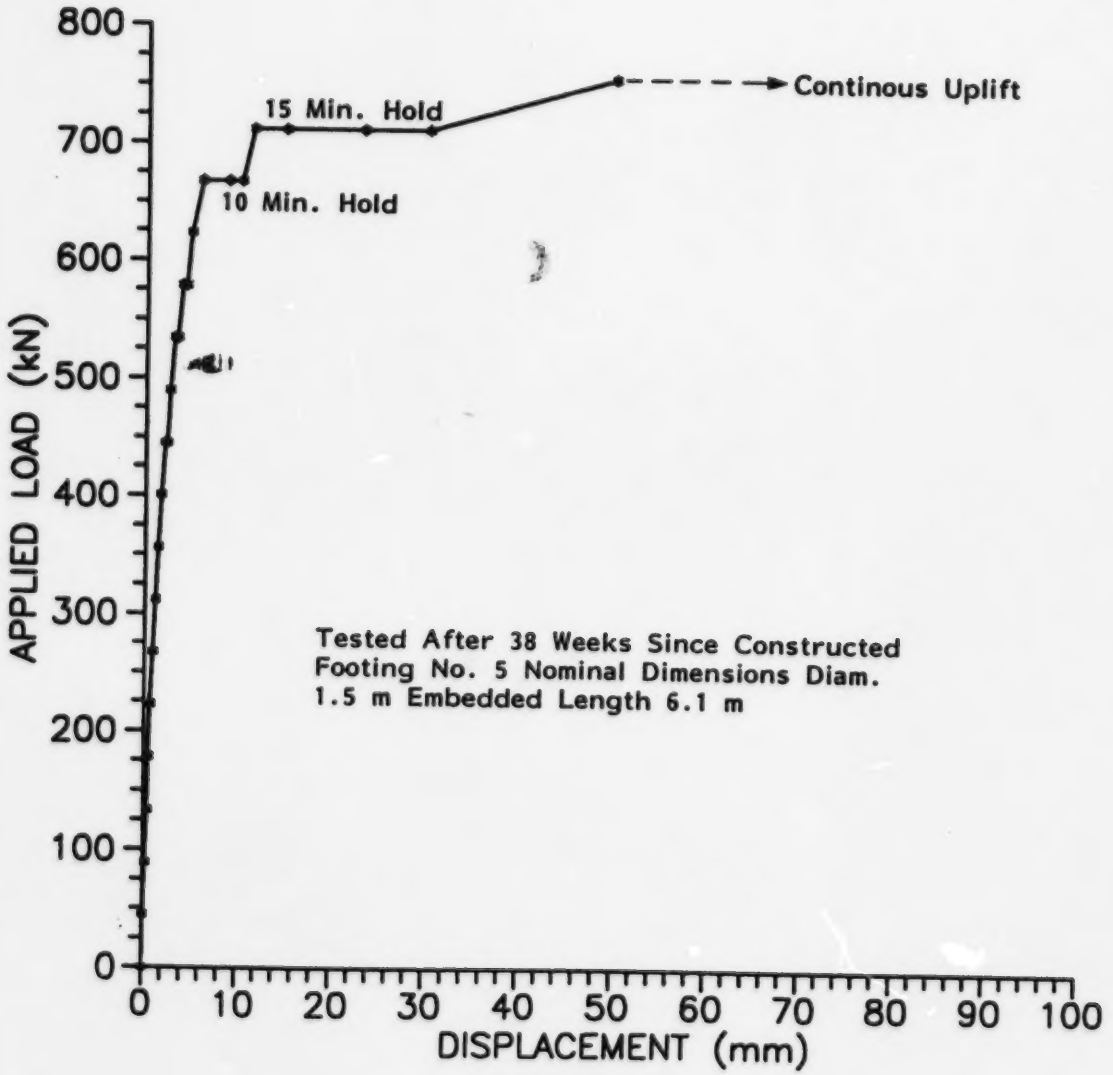
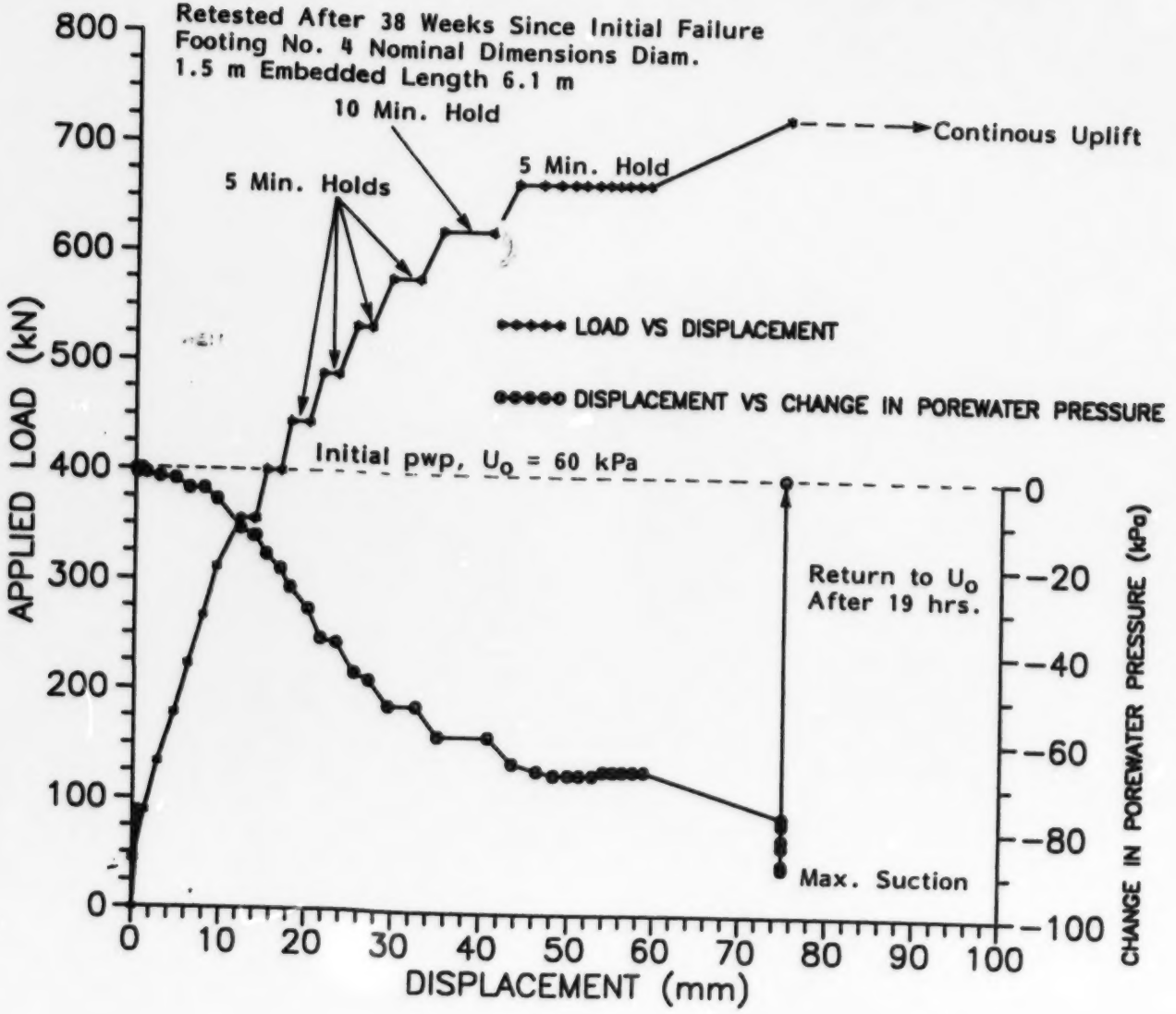


FIGURE 6

GLOUCESTER TEST SITE UPLIFT TEST OF
DRILLED PIER IN SOFT SENSITIVE CLAY



IRAD piezo it depth 7.3 m in ℄ of ftg
No change for piezo at side of ftg

FIGURE 7

GLOUCESTER TEST SITE UPLIFT TEST
OF DRILLED PIER IN SOFT SENSITIVE CLAY

APPENDIX A

CIDA-EPCRI MISSION 1219

RESEARCH DIVISION

TRANSMISSION TOWER FOUNDATIONS STUDIES

CIDA - EPCRI MISSION 1219

Aug. 29 - Sept. 9 1988

ADDENDUM TO
86-165

LIST OF PARTICIPANTS

MISSION DELEGATES

Mr. Du Zengxiang, Director of EPCRI

Madame Zheng Renping, Chief Sr. Engineer, Transmission Structure Div.

Mr. Zhang Min, Division of Technology and Economy, EPCRI

HOSTS

Dr. T.W. Klym, Manager, Civil Research

Mr. T.J. Carmichael, Supervising Engineer, Civil Research

Dr. H.S. Radhakrishna, Senior Engineer, Civil Research

Mr. C.B. Cragg, Senior Engineer, Civil Research

Dr. H.T. Chan, Senior Engineer, Civil Research

Dr. K.C. Lau, Engineer, Civil Research

Mr. A. Vorauer, Engineer, Civil Research

Mr. R.H.K. Tsang, Engineer, Civil Research

Mr. C. Pon, Engineer, Mechanical Research

Mr. G. Senkiw, Engineer, Mechanical Research

Mr. H. Dey, Design Engineer, Transmission Design Dept., Structures Group

Mr. T. Tang, Design Engineer, Transmission Design Dept., Foundation Group

Mr. G. Aitchison, Lines Superintendent, Construction

EXTERNAL EXPERTS

Mr. J.I. Adams

Mr. W. Lardner, Deep Foundations Contractors Inc.

Mr. M. Hodgson, A.B. Chance Inc.

Mr. Allan, Dywidag Canada Inc.

Dr. T. Law, Research Engineer, National Research Council of Canada

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DATE	TOPICS	SECTION
	CIDA-EPCRI MISSION 1219 Transmission Tower Foundations	A
	List of Participants	A
	Table of Contents	A
	Itinerary	B
	Questions by EPCRI	C
Aug. 29	ONTARIO HYDRO BROCHURES	D
	* Annual Report 1987 * Research Annual 1987 * Research Division - Services and Capabilities * Geotechnical Research and Development * Pocket Facts * Map of Toronto	
Aug. 30	(A) Tower Foundations	E
	(1) Loads (2) Foundations (3) Design Philosophy (4) Factors of Safety (5) Probabilistic Design (6) Cost Elements (7) Discussion	
	(B) Special Foundations	
	(1) Foundatins in Soft Clay (2) Foundations in Swamp (3) Fundex Piles (Video)	
	(C) Lateral Loads	
	(1) Steel Poles Foundations (2) Discussion	

DATE	TOPICS	SECTION
Aug. 31	Seminar by External Experts	F
	(1) Electrical Power Research Institute (Mr. J.I. Adams)	
	(2) Deep Foundations (Mr. W. Lardner)	
	(3) Anchor Systems	
	* Helix Anchors (Mr. M. Hodgson)	
	* Grouted Anchors (Mr. Allan)	
	(4) Discussion	
Sept. 1	(A) Soil Testing	G
	(1) Planning	
	(2) Reconnaissance	
	(3) Air Photo Interpretation	
	(4) Drilling Equipment	
	(5) Sampling Equipment	
	(6) In-Situ Testing	
	(7) Costs	
	(8) Discussion	
	(B) Soil Laboratory Tour	
	-Demonstration of Soil Testing	
	(1) Soil Identification	
	(2) Sample Extraction	
	(3) Strength Tests	
	(4) Atterberg Limits	
	(5) Hydrometer	
	(C) C' - O' Triaxial Test	
	(D) In-Situ Testing Devices	
Sept. 2	Meeting with Transmission Design Department	H
	(1) Structures Group (Mr. Harry Dey)	
	* Design Philosophy	
	* Loads	
	* Stress Analysis	
	(2) Foundation Group (Mr. Tom Tong)	

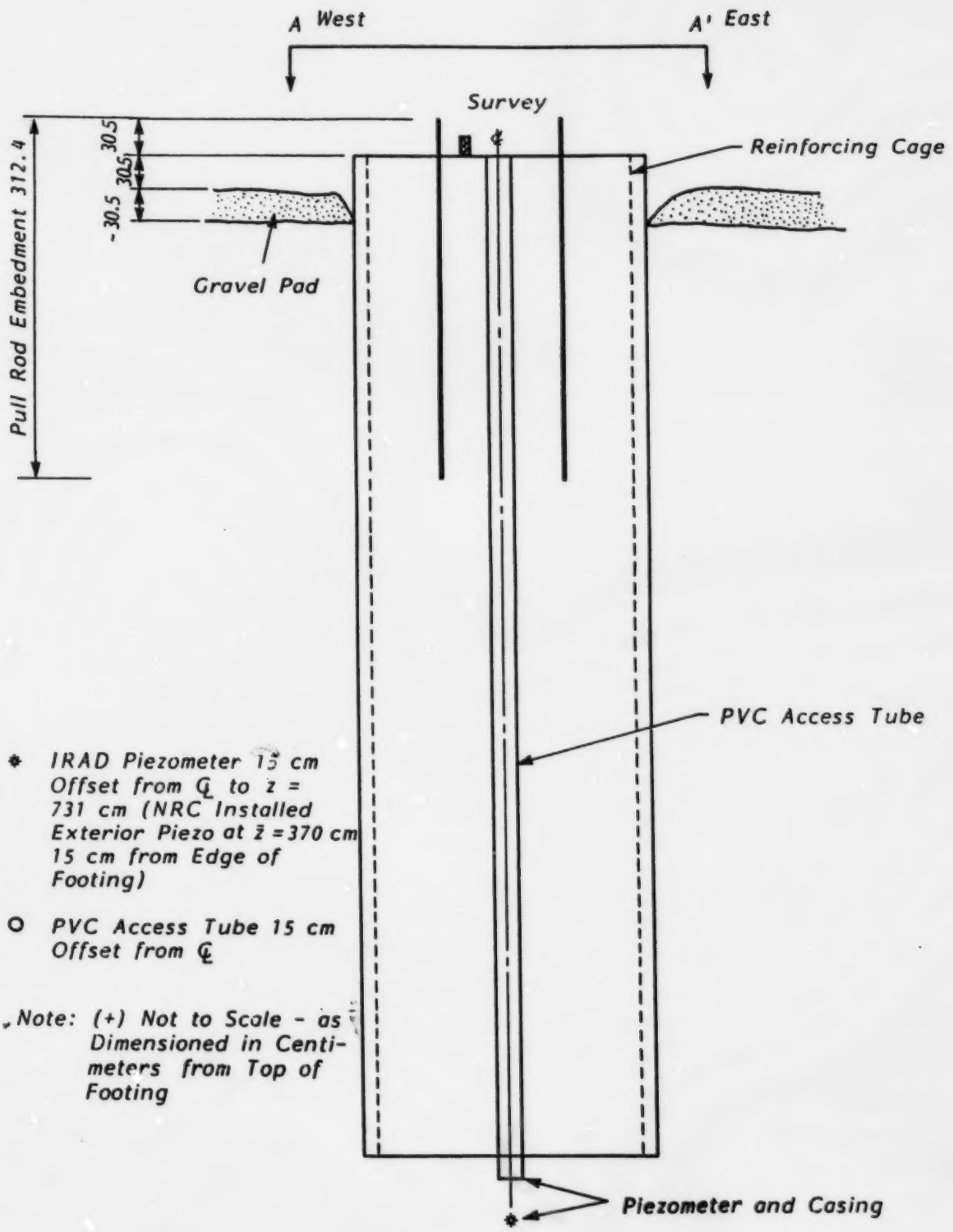
DATE	TOPICS	SECTION
Sept. 3	Day-off	J
	Shopping and sight-seeing of Toronto	
Sept. 4	Day-off	J
	Visit to Niagara Falls with Dr. H.T. Chan	
Sept. 5	Day-off	J
	* Visit Darlington G.S. * Tour of Kingston and Thousand Islands (Mr. R. Tsang and Mr. K.C. Lau)	
Sept. 6	Field Trip to Construction Site of 500 kV Transmission Line (Mr. G. Aitchison)	K
Sept. 7	Visit National Research Council of Canada (Dr. T. Law)	L
Sept. 8	(A) Full-Scale Foundation Uplift Testing in Soft Clay	M
	(1) Site Layout (2) Soil Profile (3) Schematic set-up (4) Instrumentation of Test Footing (5) Previous Uplift Test Results	
	(B) Demonstration of Uplift Load Test	
	(1) Footing #5 (2) Footing #4 (previously tested)	
Sept. 9	In-Situ Soil Testing Demonstration (Dr. Tim Law)	N
	(1) Piezocone (2) Self-Boring Pressuremeter (3) Dilatometer (4) Field Vane	
Sept. 10	Mission Leaves for Montreal at 11:00 hrs. Flight CP830 Ottawa - Montreal	O

DATE	TOPICS	SECTION
	Mechanical Research	P
	A Collection of Papers	
	(1) GALLOPING CONDUCTOR CONTROL - STATUS 1988 (D.G. Havard and C.J. Pon)	
	(2) INFLUENCE OF WIND DIRECTION ON THE PROBABILITY OF FAILURE OF TOWER FOUNDATION (S.G. Krishnasamy, C.B.H. Cragg and N.Ramani)	
	(3) PROBABILISTIC ASSESSMENT OF PENDULUMS TO CONTROL SINGLE AND BUNDLE CONDUCTOR GALLOPING (D.G. Havard, C.J. Pon and J.F. Wilson)	
	(4) LITERATURE SURVEY ON AEOLIAN VIBRATION EXCITATION OF BUNDLED CONDUCTORS (I.G. Currie, D.G. Havard and C.H. Pon)	
	(5) APPLICATION OF PROBABILITY TECHNIQUES TO TRANSMISSION LINE UPRATING (S.G. Krishnasamy, N. Ramani and J.P. Bayne)	
	(6) WIND DIRECTION AND EXTREME WIND LOADS FOR OVERHEAD POWER LINE DESIGN (S.G. Krishnasamy and N. Ramani)	
	(7) GALLOPING CONTROL PERMITS TOWER HEAD COMPACTION AND LINE UPGRADING (D.G. Havard and C.J. Pon)	
	(A) Foundation Drilling Journal	Q
	* Table of Contents * Glenwood Canyon, The Colorado River, Hard Rock, By S. Litke * Foundatio Engineering: A Viewpoint By F.H. Kulhawy	
	(B) Soil Drilling	
	* Wetson Inc. Brochures * Miscellaneous Advertisements	
	Dywidag Threadbar System	R
	-Rock and Soil Anchors	

APPENDIX B

Foundation Load Tests
Footing #4
Footing #5

(a) Instrumentation Layout
(b) Full-Scale Load Testing

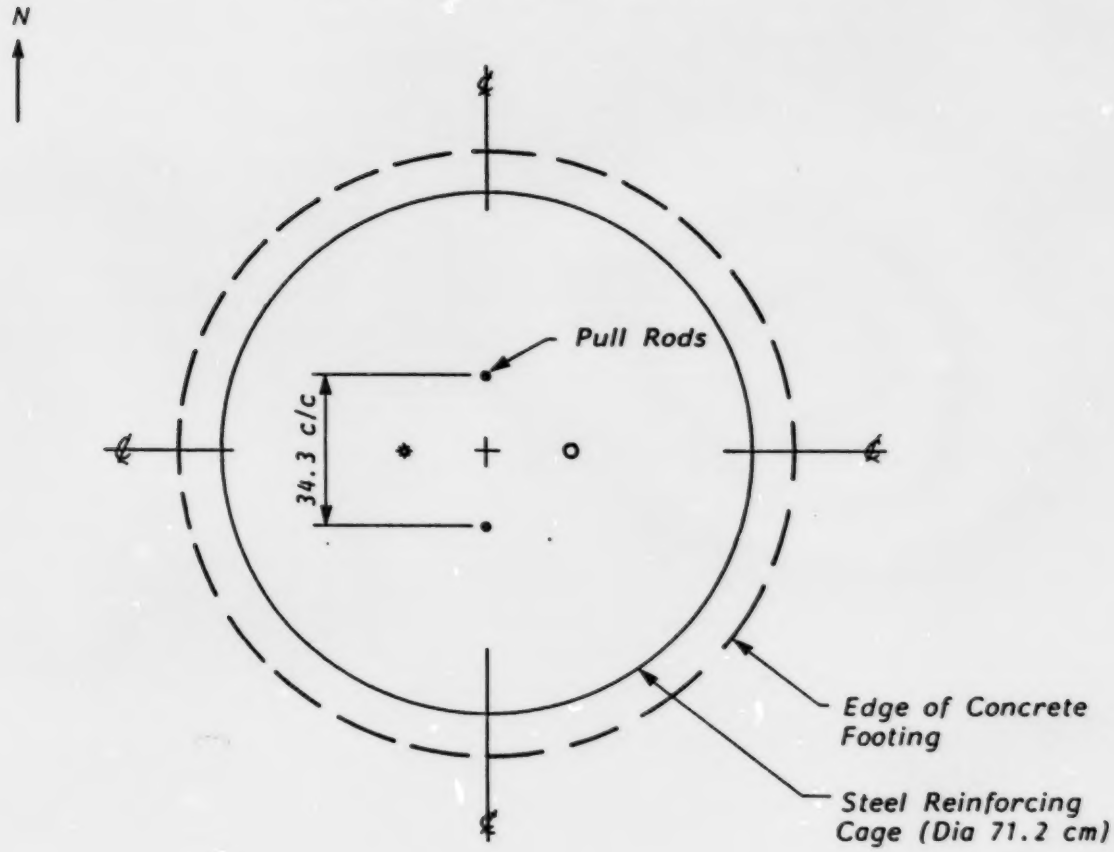

Elevation View (+)
A West
A' East
Survey
Reinforcing Cage
30.5
30.5
-30.5
Pull Rod Embedment 312.4
Gravel Pad
PVC Access Tube
IRAD Piezometer 15 cm Offset from ℄ to z = 731 cm (NRC Installed Exterior Piezo at z = 370 cm 15 cm from Edge of Footing)
PVC Access Tube 15 cm Offset from ℄
Note: (+) Not to Scale - as Dimensioned in Centimeters from Top of Footing
Piezometer and Casing

FIGURE

INSTRUMENTATION OF TEST FOOTING #4

150 m Average Diameter x 6.81 m Embedded Length ADDENDUM TO

Section A-A - Plan View (+)



* IRAD Piezometer

o PVC Access Tube

Note: (+) Not to Scale - as Dimensioned

FIGURE
INSTRUMENTATION OF TEST FOOTING #4

ADDENDUM TO

86-165

171316 RC

CIDA MISSION 1219 - FIELD LOAD DEMONSTRATION
OTTAWA - GLOUCESTER SITE

FOOTING NO. 4
DIAM.
EMBEDDED LENGTH
UPLIFT TEST ON FOOTING PREVIOUSLY TESTED TO FAILURE

APPLIED LOAD (KIPS)	VERTICAL DISPLACEMENT (INCHES)	APPLIED LOAD (kN)	VERTICAL DISPLACEMENT (mm)	CENTRE PIEZO #10802 READING	CENTRE PIEZO #10802 DELTA PWP FROM Uo (kPA)	HOLD TIME (MIN.)
0	0	0	0	3.306	0	0
10	0	44.5	0	3.306	0	
20	0.014	89	0.3556	3.306	0	
20	0.029	89	0.7366	3.306	0	4
0	0.002	0	0.0508	3.306	0	
10	0.006	44.5	0.1524	3.307	-0.83	
20	0.05	89	1.27	3.307	-0.83	
30	0.108	133.5	2.7432	3.308	-1.66	
40	0.179	178	4.5466	3.3085	-2.075	
50	0.242	222.5	6.1468	3.311	-4.15	
60	0.309	267	7.8486	3.311	-4.15	
70	0.363	311.5	9.3726	3.314	-5.64	
80	0.479	356	12.1666	3.322	-13.28	
80	0.54	356	13.716	3.324	-14.94	5
80	0.544	356	13.8176	3.324	-14.94	10
80	0.551	356	13.9954	3.324	-14.94	15
90	0.599	400.5	15.2146	3.329	-19.09	
90	0.665	400.5	16.891	3.333	-22.41	5
100	0.709	445	18.0086	3.338	-26.56	
100	0.792	445	20.1168	3.344	-31.54	5
110	0.849	489.5	21.5646	3.352	-38.18	
110	0.919	489.5	23.3426	3.353	-39.01	5
120	0.999	534	25.3746	3.361	-45.65	
120	1.064	534	27.0256	3.363	-47.31	5
130	1.154	578.5	29.3116	3.37	-53.12	
130	1.281	578.5	32.5374	3.37	-53.12	5
140	1.379	623	35.0266	3.378	-59.76	
140	1.604	623	40.7416	3.378	-59.76	5
150	1.714	667.5	43.5356	3.385	-65.57	
150	1.824	667.5	46.3296	3.387	-67.23	1
150	1.899	667.5	48.2346	3.389	-68.06	2
150	1.964	667.5	49.8856	3.388	-68.06	3
150	2.014	667.5	51.1556	3.388	-68.06	4
150	2.074	667.5	52.6796	3.388	-68.06	5
150	2.124	667.5	53.9496	3.387	-67.23	6
150	2.169	667.5	55.0926	3.387	-67.23	7
150	2.214	667.5	56.2356	3.387	-67.23	8
150	2.264	667.5	57.5056	3.387	-67.23	9
150	2.314	667.5	58.7756	3.387	-67.23	10
164	)3	729.8	75	3.399	-77.19	
164	)3	729.8	75	3.405	-82.17	1
164	)3	729.8	75	3.413	-88.81	2
164	)3	729.8	75	3.412	-87.98	3
164	)3	729.8	75	3.407	-83.83	4
0	UNLOAD	0		3.407	-83.83	5
0		0		3.401	-78.85	30
0		0		3.4	-78.02	37
0		0		3.306	0	1140

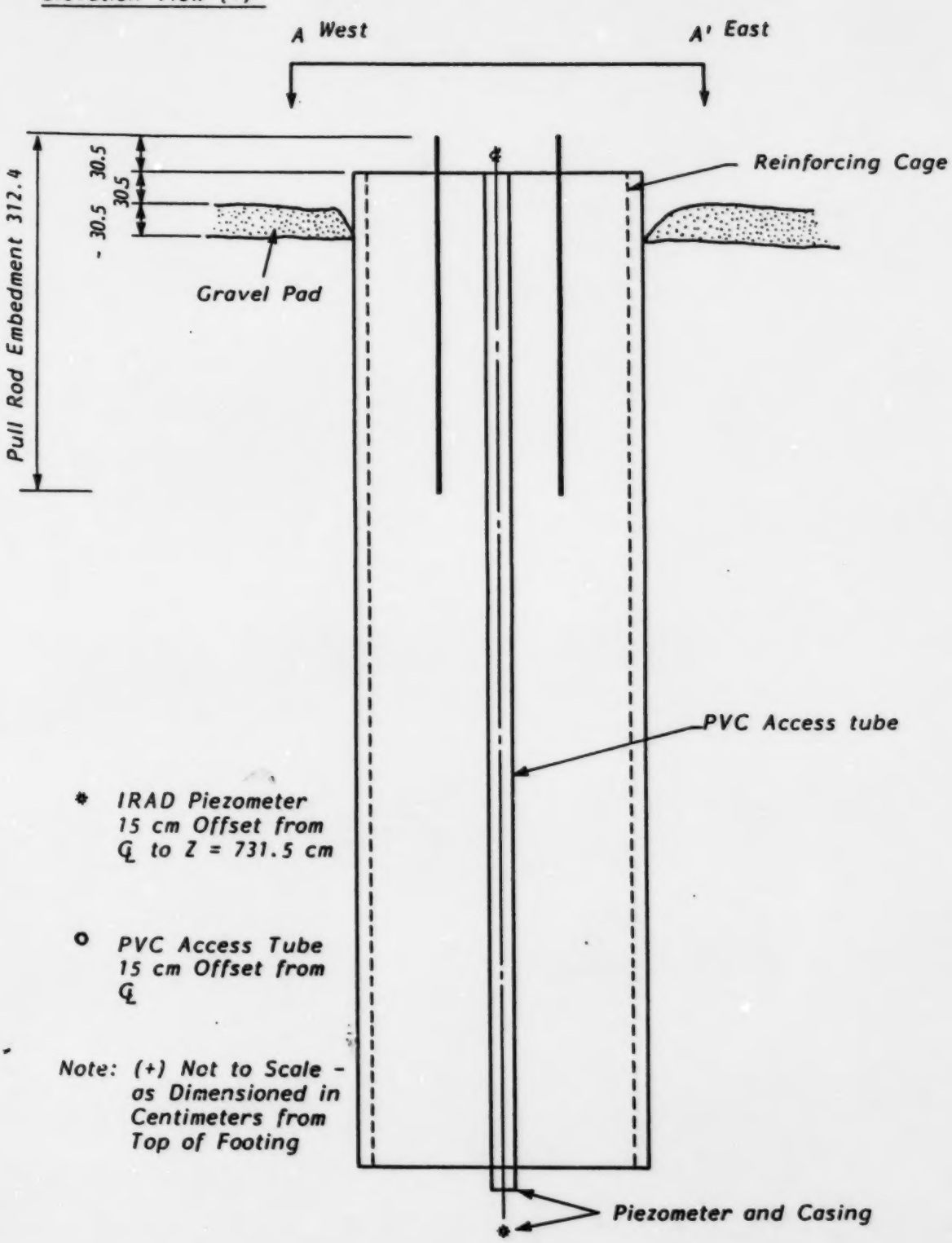
NOTE (1) ASSUME INITIAL POREWATER PRESSURE, Uo = 60 kPA

IRAD PIEZOMETER NO. 10802 AT DEPTH z = 731 cm IN CENTRE OF FOOTING
IRAD PIEZOMETER NO. 10816 AT DEPTH z = 370 cm AT THE EDGE OF FOOTING

IRAD PIEZOMETER NO. 10816 ESSENTIALLY NO CHANGE IN POREWATER PRESSURE THROUGHOUT THE TEST

CALIBRATION CONSTANT FOR #10802 (0.01 = 8.3 kPA)
CALIBRATION CONSTANT FOR #10816 (0.01 = 10.24 kPA)

READOUT DEVICE SETTINGS MODEL MB-6-LU
S/N 0181
LINEAR SETTING
AUTO SETTING
GAUGE TYPE - PWP



FIGURE

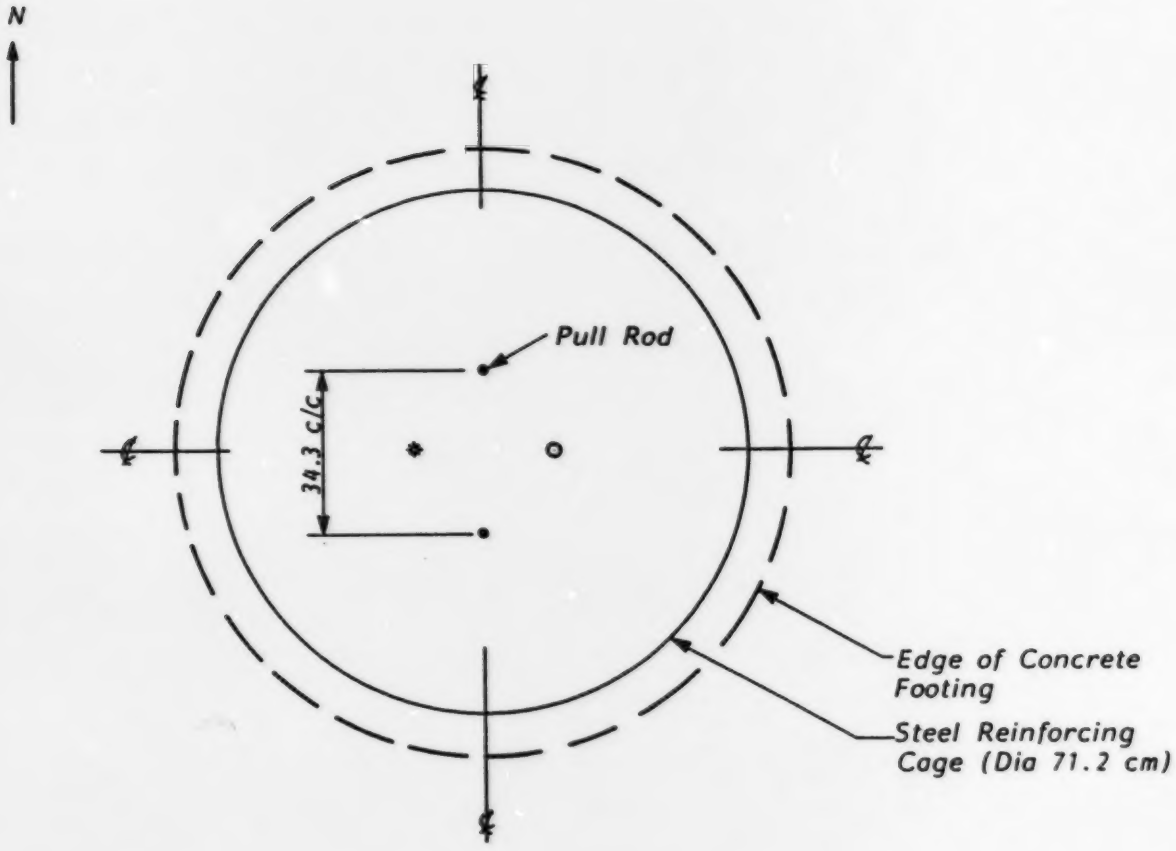
INSTRUMENTATION OF TEST FOOTING #5

150 m Average Diameter x 6.32 m Embedded Length

ADDENDUM TO
86-165

171317 RC

Section A-A' - Plan View (+)



* IRAD Piezometer

o PVC Access Tube

Note: (+) Not to Scale - as Dimensioned

FIGURE

INSTRUMENTATION OF TEST FOOTING #5

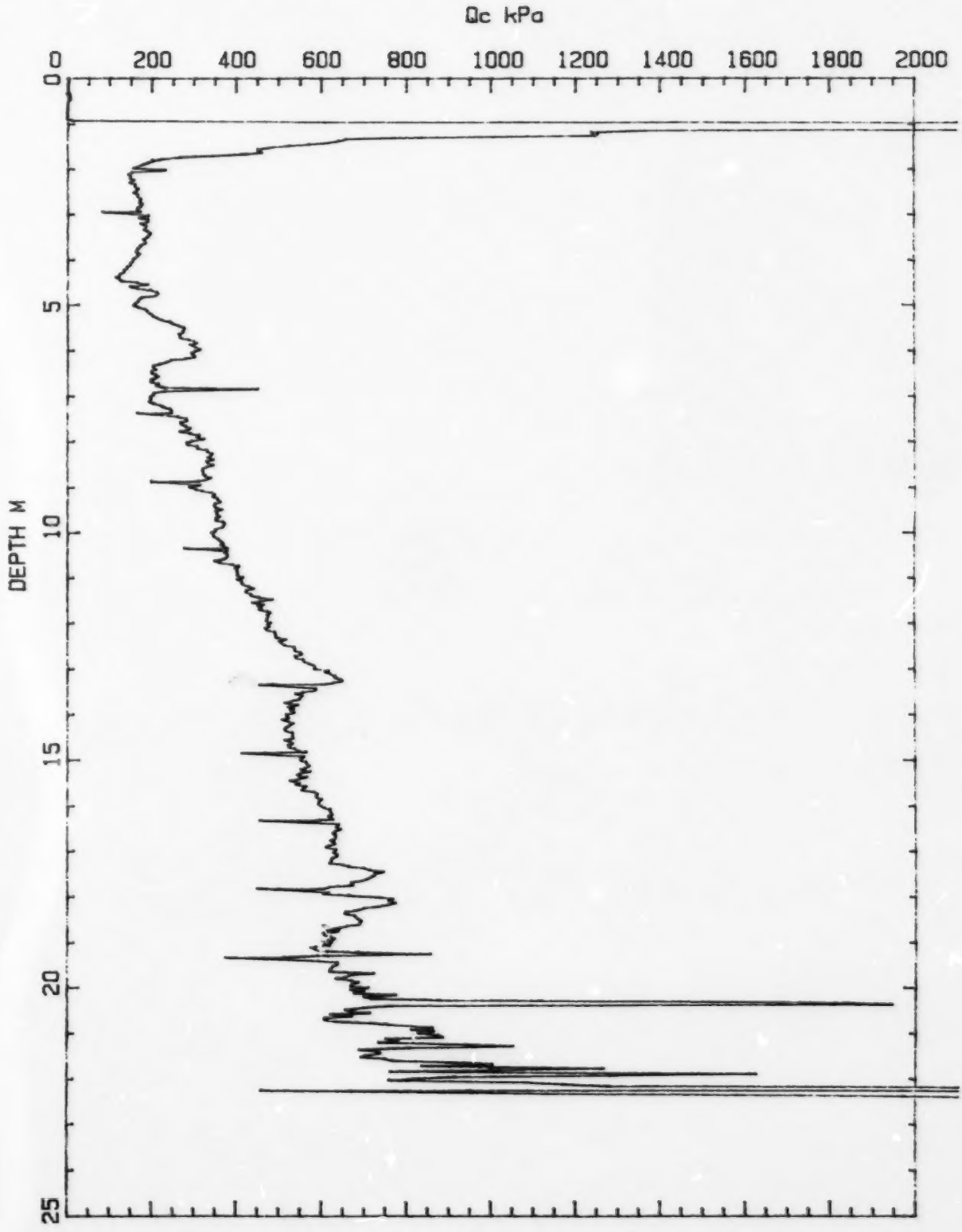
CTTAWA - GLOUCESTER SITE

FOOTING NO. 5
DIAM.
EMBEDDED LENGTH
UPLIFT TEST

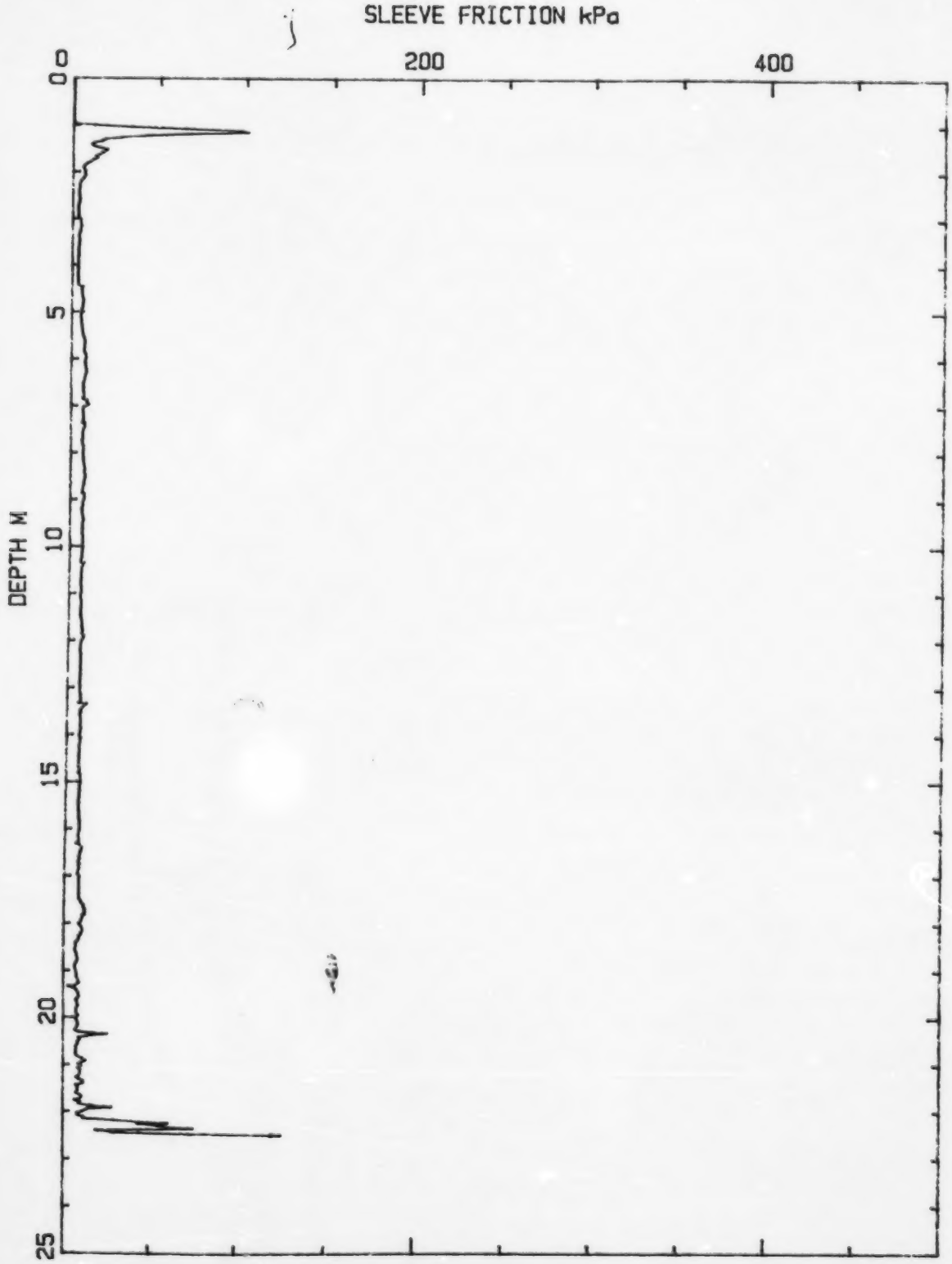
APPLIED LOAD (KIPS)	VERTICAL DISPLACEMENT (INCHES)	APPLIED LOAD (kN)	VERTICAL DISPLACEMENT (mm)	HOLD TIME (MIN.)
0	0	0	0	0
10	0.002	44.5	0.0508	
20	0.008	89	0.2032	
30	0.015	133.5	0.381	
40	0.018	178	0.4572	
50	0.025	222.5	0.635	
60	0.035	267	0.889	
70	0.044	311.5	1.1176	
80	0.053	356	1.3462	
90	0.065	400.5	1.651	
100	0.08	445	2.032	
100	0.088	445	2.2352	5
110	0.099	489.5	2.5146	
120	0.117	534	2.9718	
120	0.13	534	3.302	
130	0.148	578.5	3.7592	
130	0.166	578.5	4.2164	
140	0.185	623	4.699	
150	0.23	667.5	5.842	
150	0.343	667.5	8.7122	5
150	0.4	667.5	10.16	10
160	0.45	712	11.43	
160	0.59	712	14.986	5
160	0.915	712	23.241	10
160	1.19	712	30.226	15
170	>1.25	756.5	50	CONTINOUS UPLIFT TO 2 INCHES

APPENDIX C

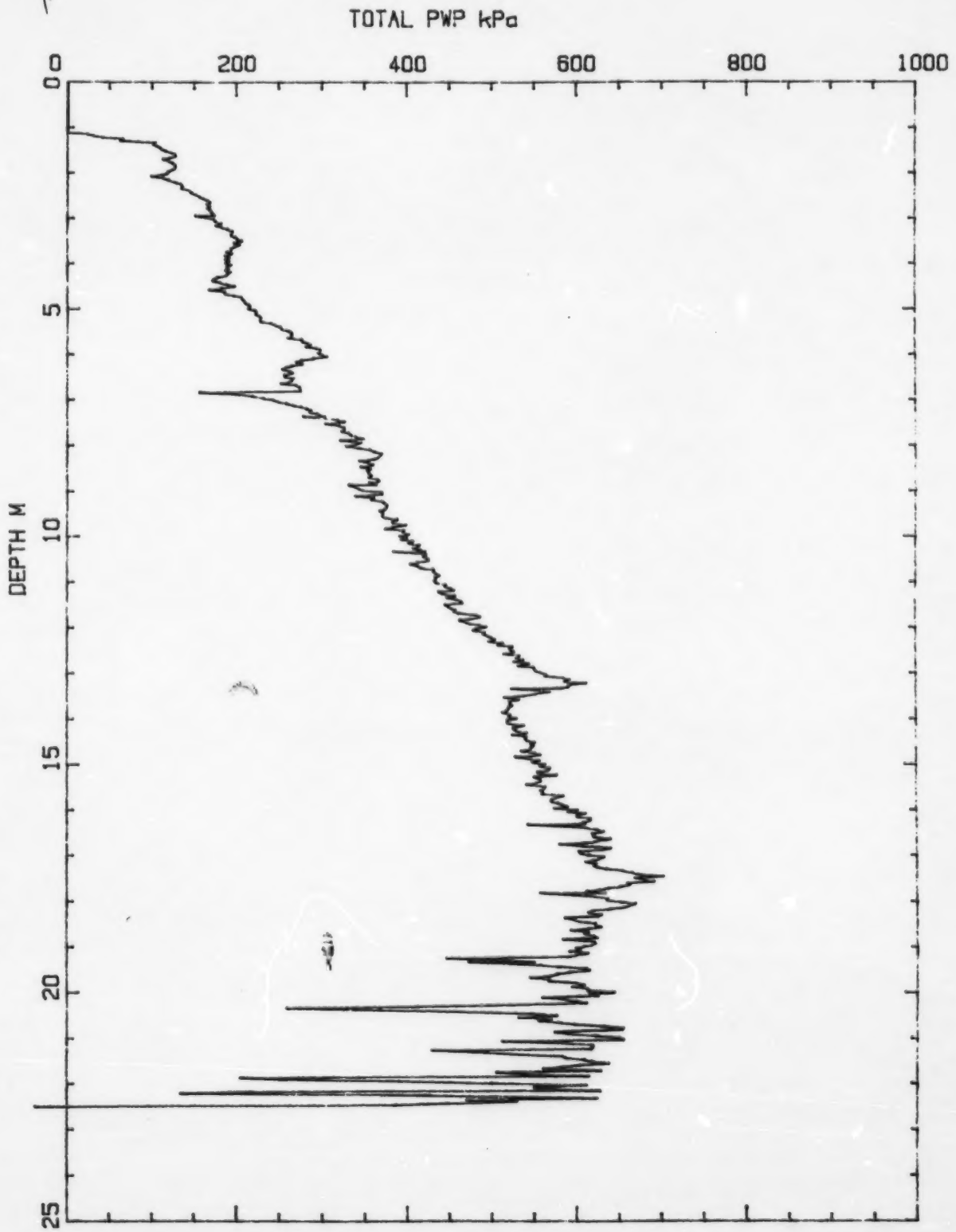
Piezocone Profile of Undrained Shear Strength
by National Research Council



GLOUCESTER 88 C1



GLOUCESTER 88 C1 (DEMONSTRATION FOR EPCRI)



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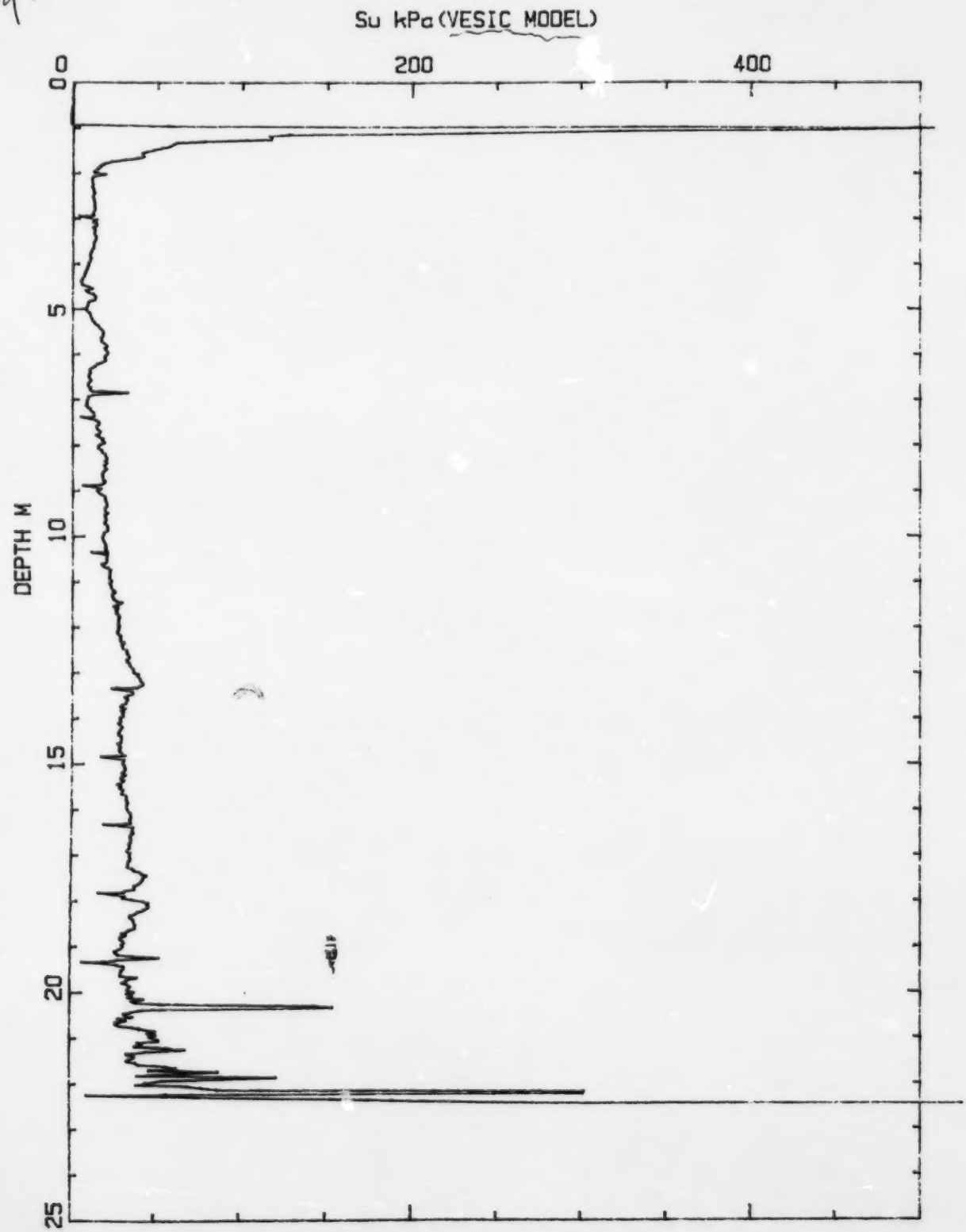

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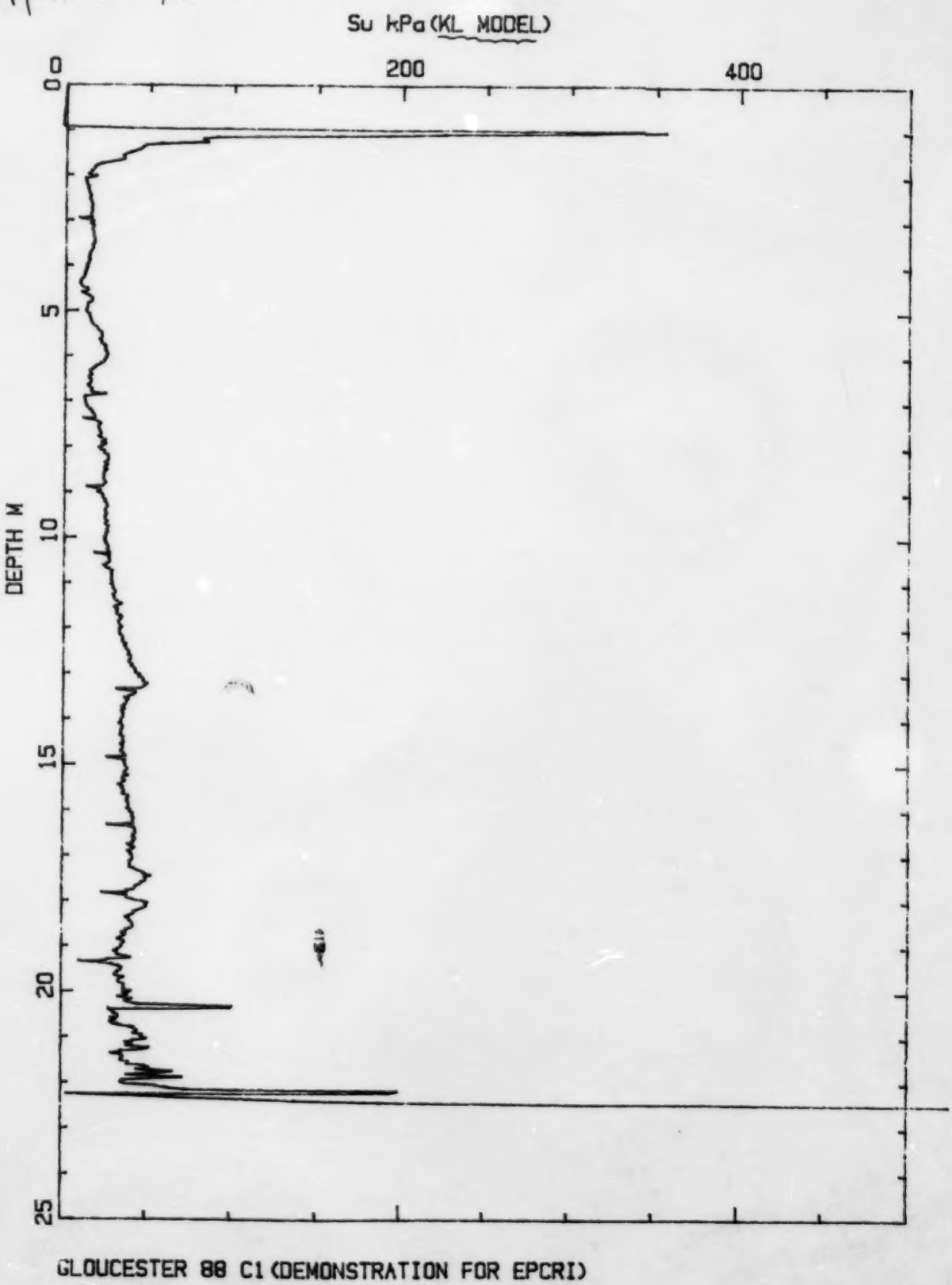

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